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September 14, 2016

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Supplemental Recommendations Post-Tensioned Slabs-on-Ground Foundations
THE GROVE RESIDENTIAL SUBDIVISION
3342 Humphrey Road
Loomis, California
WKA No. 11071.01

We understand that the development of the proposed site of *The Grove* residential subdivision located at 3345 Humphrey Road in Loomis, California is considering the use of post-tensioned slabs for the foundations. This letter presents geotechnical design recommendations to support design of the post-tensioned slab foundations. The recommendations found in this letter are supplemental recommendations to the original Geotechnical Engineer Report (GER) dated September 9, 2016 and should be used in conjunction with those recommendations when developing the final foundation design, plans, and specifications.

Post-tensioned slab foundations are commonly used when subgrade soils consist of expansive clay material. However, soils at this site are considered to be relatively non-expansive. We understand that although post-tensioned slab foundations are not required for geotechnical reasons that they may be selected based on other design and construction considerations.

Should post-tensioned slab foundations be selected for use at this site, a qualified post-tensioned slab designer should be contacted directly to provide minimum design parameters (i.e. slab thickness, reinforcement, edge lift, center lift, etc.) acceptable by the Post-Tensioning Institute for post-tensioned foundation slabs supported on stable, non- to low expansive soils. Geotechnical information for use in the design of post-tensioned slabs is presented below.

Post-Tensioned Slab Design Recommendations

The Third Edition of the Post-Tensioning Institute's "Design of Post-Tensioned Slabs-on-Ground" manual (2004) contains structural guidelines for design of post-tensioned slab foundations on stable, non- to low-expansive soils (see Section 2.0). The site of *The Grove* residential subdivision does not have excessive expansive or compressible soils, and a Building Research Advisory Board Repot (BRAB) Type II foundation or alternate foundation such as a spread and/or strip footings may be utilized (see section titled *Foundation Design* in original GER).

Building pads for support of PT Slab foundations should be prepared using the recommendations contained in the geotechnical report. PT Slab foundations should be designed for an allowable soil bearing pressure of 2,500 pounds per square foot (psf) for the dead plus live load condition. The allowable post-tensioned slab bearing capacity may be increased by 1/3 to include wind or seismic forces.

Resistance to lateral foundation displacement may be computed using an allowable friction factor of 0.35 for the portion of the slab in contact with compacted fill or rock, or 0.25 for the portion of the slab in contact with sand or pea gravel over the vapor retarder membrane. The friction factors may be multiplied by the effective vertical load on that portion of the foundation. Additional lateral resistance may be computed using an allowable passive earth pressure of 400 psf per foot of depth of embedment below the compacted soil surface. These two modes of resistance should not be added unless the frictional value is reduced by 50 percent since full mobilization of these resistances typically occurs at different degrees of horizontal movement.

Friction factors for determining effective strand forces, should be determined based on Section 2.2 of the PT Slab Design manual.

The most common post-tensioned slab foundations used in the greater Sacramento area are 8- to 10-inch thick slabs, thickened to 10 to 12 inches at the perimeter; however, the slab designer should determine the final slab thickness.

Post-tensioned foundation slabs should be underlain by a durable vapor retarder (at least 10 mils thick) placed directly on the final soil subgrade, covered with an "optional" two inches of damp, clean sand, or as recommended by the slab designer. The soils below the membrane should be at the optimum moisture content or wetter at the time of foundation construction.

Limitations

Our recommendations are based upon the information provided regarding the proposed construction, combined with our analysis of site conditions revealed by the field exploration and laboratory testing programs discussed in the original GER dated September 9, 2016. This letter is considered an addendum to the GER and is therefore subject to the limitations stated in the report.

If you have any questions regarding this letter, or with any other geotechnical engineering aspects of the project, please contact our office.

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Geotechnical Engineering Report
THE GROVE RESIDENTIAL SUBDIVISION
WKA No. 11071.01
September 9, 2016

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Geotechnical Engineering Report
THE GROVE RESIDENTIAL SUBDIVISION
3342 Humphrey Road
Loomis, California
WKA No. 11071.01
September 9, 2016

INTRODUCTION

We are pleased to present this geotechnical engineering report for the proposed site of *The Grove* residential subdivision located at 3345 Humphrey Road in Loomis, California as shown on Figure 1. The purposes of our study has been to characterize the existing site subsurface conditions and develop engineering conclusions and recommendations with regards to the design and construction of the proposed single family home housing development. This report presents the results of our study.

Scope of Services

Our scope of services for this project has included the following tasks:

- Data review, including review of previous studies and available historical and geologic data available at or near the site.
- Site Reconnaissance, including observation of existing site conditions and marking for Underground Service Alert.
- Site investigation, including a field investigation consisting of excavation of seven test pits and drilling of six soil borings in order to sample and log subsurface conditions.
- Laboratory testing program, completed to characterize the in-situ subsurface conditions and engineering properties.
- Engineering analysis, completed to evaluate geotechnical properties and seismic hazards.
- Development of recommendations, including design and construction recommendations regarding 2013 CBC seismic criteria, site grading, drainage, foundations, retaining walls, and pavement sections. Also included are preliminary earthwork specifications.
- Presentation of findings, conclusions, and recommendations in this design level geotechnical engineering report (GER).

In addition to the above services, we anticipate that consultation and plan review services will be provided as needed to support the preparation of the plans and specifications during the design and review phases of the project.

Figures and Attachments

This report contains a Vicinity Map as Figure 1, and a Site Plan as Figure 2. Figures 3 through 8 contain the logs of borings and Figure 9 and 10 contain the logs of test pits. Figure 11 contains an explanation of the symbols and classification system used on the boring and test pit logs.

Appendix A contains general information regarding the project, the field exploration and laboratory testing program, including test results not presented on the boring and test pit logs

Appendix B contains *Earthwork Specifications* for use in preparing contract documents.

Proposed Development

Review of the *Proposed Lotting Exhibit* provided by Meredith Engineering and dated August 3, 2016 for *The Grove* development indicates that the project will consist of 26 lots. Lots 1 - 22 will be intended for single family home construction, while lots A – C are intended for use as public landscape, and lot D is intended for use as a storm water detention basin.

We anticipate the residential structures will be one- to two-story, wood-framed structures with interior concrete slab-on-grade lower floors. Structural loads are anticipated to be relatively light, consistent with this type of construction. Associated development will include underground utilities, exterior flatwork, new residential streets, retaining walls, a storm water detention basin and typical landscaping.

The Conceptual Grading and Drain Plan prepared by Meredith Engineering (undated) indicate the final finished grades for the building pads will vary from about +365 to +373 feet. Based on the topography we anticipate that excavations (cuts) and fills on the order of one to eight feet will be required to develop the property, with the deepest section of fill located within Lot 8 at the southwest corner of the subdivision. The estimated fill depths do not include excavation depths for underground utility improvements or drainage.

FINDINGS

Site Description

The site is located at 3342 Humphrey Road in Loomis, California. The site is roughly rectangular in shape, and is bounded on the north and east sides by No Name Lane and

Humphrey Road, respectively. Single story homes north of Myrtle Drive bound the southern edge of the site, while the western edge is bounded by single story residential units fronted by an unimproved private roadway. The site area encompasses a gross area of approximately 9.98 acres, identified as parcel (APN) 044-021-008-0000.

At the time of our site reconnaissance performed on August 1, 2016 the site was observed to be vacant and covered with light to moderate growth of weeds and grass, and mature trees. The site is surrounded by an existing wire fence, with a gate opening onto Humphrey road. Existing utility distribution poles, running north to south, lie along the eastern edge of the site. Debris, including old metallic fence posts, were observed on the surface and an open ended pipe was observed in the north west corner of the site near the existing wetland pond.

A survey of the site was completed on June 10, 2016 by Andregg Geomatics. Based on this survey, topography of the site is expected to vary from a minimum elevation[1] of about 359 feet in the southwest corner of the site to a maximum elevation of about 375 feet along the east edge of the site near Humphrey Road.

Seasonal swales, a seasonal pond, and a seasonal wetland were reported by Andregg Geomatics in their survey from 2016. Surface features consistent with the reported swales and wetlands/pond were observed during site activities along the north edge of the property and in the southwest corner. Based on the reported extent of these features, the northern swale passes under the location of Lots 13 through 15 and drains towards the seasonal pond (e.g. location of proposed drainage pond at Lot D). We observed a ditch extending from the eastern end of the swale towards lot A (e.g. under Lots 15 through 17) with the eastern end of the ditch extending to a small basin located at the boundary of Lot 17 and Lot A. The second swale identified by Andregg Geomatics is located in the southwest corner of the site, and drains towards and through Lot 8. The end of this swale currently drains to the meandering drainage easement which is located just to the south and west of the site, as identified by Meredith Engineering on their drawing dated August 9, 2016 titled *Proposed Lotting Exhibit*.

<u>Site History</u>

We reviewed historical areal and satellite imagery, and past reports of the site history available at the time of this report. Documents reviewed included reports by Soil Search Engineering (2005) and Earthtec, Ltd (2005) review of the Department of Toxic Substances Control site history, and review of areal and satellite imagery available in Google Earth.

[1] All elevations reported are provided with respect to the North American Vertical Datum of 1988 (NAVD88). Reported elevations in this report are approximate, and are generally based on topographic data from the Andregg Geomatics survey completed June 10, 2016.

Based on our review, the site was originally used as an orchard, with limited cattle grazing. A residence and supporting structures were located near the existing gate on the eastern edge of the property were removed prior to 2002. The site has reportedly lain fallow since 1961 with excavated soil from a nearby pool excavation spread along portions of the southern boundary of the site in 2004. Based on review of historical areal and satellite imagery available from Google Earth for the years from 1993 to 2015, the site has remained vacant in recent years, with no significant use or engineered structures on the site during this timeframe.

Geology

The site lies near the boundary of the Great Valley geomorphic province, which is an alluvial plain about 50 miles wide and 400 miles long, and the Sierra Nevada geomorphic province in the vicinity of what is known locally as the Penryn and Rocklin plutons. Regional geology in the vicinity of the site is characterized by the United States Geological Society (USGS, 2007) as Mesozoic granitic rocks (grMz). The Mesozoic granitic rocks generally consist regionally of Mesozoic granites, quartz monzonite, granodiorite, and quartz diorite.

Surface soils are characterized by the US department of agriculture (USDA) as Andregg coarse sandy loam (USCS Silty Sand, SM), with a typical soil profile of 29 inches of Andregg sandy loam over weathered bedrock. These soils are classified by the USDA as moderately corrosive.

Subsurface Soil Conditions

The near-surface soil and rock stratigraphy were observed using exploratory test pits and soil borings, located in the approximate locations shown on Figure 2. Site soil and rock stratigraphy generally consisted of a layer of silty sand (SM) overlying weathered bedrock (RX) to the maximum depth explored. We observed a boulder or outcrop of granodiorite exposed at the surface in the southern portion of the site. Organics and rootlets were present across much of the site to depths of about 3 feet below the ground surface, with larger concentrations of organics in the upper 6 to 12 inches.

The silty sand (SM) was composed of a brown to grayish or reddish brown variably cemented silty fine to coarse sand which extended to depths ranging from about 2 to 5 feet below the ground surface. We note that the upper six to twelve inches of the silty sand were generally lightly disturbed and less cemented than deeper portions of the soil layer.

The silty sand was underlain by weathered or decomposed granitic (granodiorite) bedrock (RX), to the maximum depth explored. During excavation of the test pits, the weathered bedrock was observed to break down, primarily into silty fine to coarse sand (SM). Limited clay seams were observed at depth in the weathered bedrock in Test Pit 7. The weathered bedrock generally becomes less weathered and increasingly difficult to excavate with depth.

Depth to practical refusal will vary with equipment type, effort, and location. During exploratory activities practical refusal was reached at depths of between about 5 and 12.5 feet below the ground surface. We anticipate that using typical excavation equipment, and varying amounts of effort, excavations of between 5 to 7 feet should be achievable across the majority of the site. Some areas where harder bedrock is exposed as outcrops or boulders may require greater effort including blasting or breaking.

For specific information regarding the subsurface conditions at a specific location, please refer to the Logs of Soil Borings (Figures 3 through 8) and the Logs of Test Pits (Figures 9 and 10).

Groundwater

Groundwater was observed in three of the soil borings (D1, D3, and D4) at the time of drilling, and ranged in depth between 8 and 13 feet below the ground surface. This corresponds to groundwater elevations of between +356 feet and +359 feet (NAVD88). Please note that the groundwater encountered does not likely represent static equilibrium conditions as the borings and test pits were only open for a short time.

CONCLUSIONS

Seismicity and Faults

The Alquist-Priolo (AP) Earthquake Fault Zoning Act regulates activities near active faults within what is known as an earthquake fault zone. Active faults are defined as a fault that has ruptured in the last 11,000 years. Review of the AP Maps of California, prepared by the California Geological Survey (CGS), shows that there are no mapped active faults in the vicinity of the site.

In our opinion, low to moderate amounts of ground shaking can occur at the site. The average peak ground acceleration (PGA_M) at the site for the maximum considered earthquake (MCE) event (per ASCE 7-10) is 0.187 g. Faults contributing significantly to the hazard include the Hunting Creek – Berryessa Fault located approximately 60 miles west of the site, and the Great

Valley fault system with significant contributions from sections lying approximately 40 to 50 miles from the site. Other, closer faults contribute less significantly to the overall hazard. These include the Foothills Fault System, the western edge of which lies approximately 7 to 8 miles east of the site, which has historically experienced higher levels of seismicity with magnitude 5 to 6 events occurring in 1888, 1909, and 1975 on more distant portions of the fault system. No known active faults cross through the site and fault displacement hazard is considered to be negligible at the site.

2013 California Building Code/ASCE 7-10 Seismic Design Criteria

Code based seismic design parameters were developed for the site consistent with section 1613 of the 2013 edition of the California Building Code (2013 CBC), which references the American Society of Civil Engineers Standard 7-10 (ASCE 7-10).

Review of the geology and site explorations shows that the majority of the soil and rock in the upper 100 feet (30 m) consists of bedrock, with the upper observed portion consisting of predominantly of weathered to decomposed granitic rock. In accordance with chapter 20 of ASCE 7-10, softer and more highly fractured and weathered rock shall be classified as a Site Class C in the absence of in-situ shear wave velocity measurements. In our opinion, Site Class C most closely approximates the conditions at this site.

Seismic design parameters were obtained for a Site Class C site using the online USGS *US Seismic Design Maps* application. Using this tool, values were obtained for a point near the center of the site located at latitude 38.8296 and longitude -121.2023, and are provided in Table 1. These seismic design parameters may be used for seismic design of the proposed residential subdivision.

TABLE 1
2013 CBC/ASCE 7-10 SEISMIC DESIGN PARAMETERS

Latitude: 38.8296° N Longitude: 121.2023° W	ASCE 7-10 Table/Figure	2013 CBC Table/Figure	Factor/ Coefficient	Value
Mapped MCE_R S_a, 0.2 sec	Figure 22-1	Figure 1613.3.1(1)	S_S	0.477 g
Mapped MCE_R S_a, 1.0 sec	Figure 22-2	Figure 1613.3.1(2)	S_1	0.242 g
Soil Class	Table 20.3-1	Section 1613.3.2	Site Class	C
Site Coefficient	Table 11.4-1	Table 1613.3.3(1)	F_a	1.200
Site Coefficient	Table 11.4-2	Table 1613.3.3(2)	F_v	1.558
Transition Period	Figure 22-12	-	T_L	12 sec
Adjusted MCE_R S_a	Equation 11.4-1	Equation 16-37	S_{MS}	0.572 g

TABLE 1 2013 CBC/ASCE 7-10 SEISMIC DESIGN PARAMETERS				
Parameters	Equation 11.4-2	Equation 16-38	S_{M1}	0.377 g
Design Level S_a Parameters	Equation 11.4-3	Equation 16-39	S_{DS}	0.381 g
	Equation 11.4-4	Equation 16-40	S_{D1}	0.251 g
Seismic Design Category	Table 11.6-1 and Table 11.6-2	Table 1613.3.5(1) & Table 1613.3.5(2)	-	D

NOTES:

MCE_R = Risk-Targeted Maximum Considered Earthquake

S_a = Five percent damped spectral response acceleration (for a given period)

g = gravity

sec = seconds

Liquefaction and Seismic Compression

A detailed evaluation of seismic compression and liquefaction effects, such as settlement and strength loss, were beyond the scope of the current study. However, a brief review of the geology and other site characteristics was made with respect to potential for detrimental seismic effects. Based on this review it is our opinion that the silty sand is unlikely to undergo significant amounts of settlement or seismic compression; in part due to the cementation of the soil particles, seasonal fluctuation of the water table, and the potential for reworking and excavation of these soils during site preparation. In our opinion the weathered bedrock exposed at relatively shallow depths across the site is not susceptible to liquefaction or seismic compression and has negligible risk.

Based upon the known geologic conditions, it is our opinion that the potential for detrimental seismic compression or liquefaction effects at the site affecting life safety is low.

Bearing Capacity and Settlement

In our opinion, the native undisturbed soils, weathered rock, and harder rock present at the site are capable of supporting the residential structures, retaining walls and other improvements planned for this site. Existing disturbed soils or soils disturbed by demolition activities are not considered suitable for support of the planned improvements unless the materials are reworked as engineered fills in accordance with the recommendations of this report.

Engineered fill composed of native soils or approved imported fill materials, properly placed and compacted in accordance with the recommendations of this report, should provide adequate support for the proposed structures and other site improvements. Specific recommendations to over-excavate, scarify, moisture condition, and recompact the surface soils are provided in the *Subgrade Preparation* section of this report.

In order to reduce the potential for damaging differential settlements under the residential structures and other improvements, a higher degree of compaction will be recommended for deeper fills and within areas of the site that will receive significant differential depths of engineered fill. Foundations spanning across soil to weathered to hard rock also should be avoided due to the potential for different elastic performance of the differing materials under transient loads.

Assuming the recommendations and conditions discussed in this report are followed, we anticipate total settlements under static conditions will be limited to less than 1 inch, and differential settlements will be limited to less than ½ inch within the footprint of the individual housing units.

Material Suitability

In our opinion, the on-site surface and near-surface soils are considered suitable for use as general fill materials provided they are appropriately processed such that they are free of debris, significant clay concentrations, are at a moisture content that will allow the recommended compaction, and contain less than 2 percent by weight of organics.

Weathered granodioritic rock is considered suitable for use as general fill provided it is suitably processed as described above, and is mechanically broken down such that it degrades to a silty sand (SM). Unweathered rock, when encountered, would only be suitable for use as fill if it can be processed into pieces no larger than 3 inches in maximum dimension, and mixed with a sufficient amount of soil to allow for a compactable mixture of soil and rock.

Soil Expansion Potential

Based on the results of our field investigation and laboratory testing, the surface and near-surface soils consist primarily of granular soils that are considered to be relatively non-expansive. Therefore, special site preparation or foundation designs to mitigate expansive soils are not considered necessary for development of this site.

Excavation Conditions

The surface and near-surface soils and highly weathered granodioritic rock are anticipated to be excavatable with conventional excavation equipment. Cuts within these soils are expected to be relatively stable at near-vertical inclinations for the short time required for foundation and utility construction. We note, however, that if soil or rock are saturated, or are subjected to induced loads, or vibrations, collapse may occur.

Excavations exceeding five feet in depth that will be entered by workers will require shoring, bracing, sloped excavations, or the use of a traveling shield conforming to current California Occupational Safety and Health Administration (Cal/OSHA) regulations. Generally, we anticipate that soil excavations will be designed consistent with an OSHA Type B soil for unsaturated conditions, and an OSHA Type C for saturated conditions, or if freely seeping. We recommend that weathered granodioritic rock should NOT be considered "stable rock" due to the severity of the weathering and potential for collapse. Final design and oversight of temporary excavation slopes should be made the responsibility of the Contractor, since the Contractor is on site and may employ a competent person to observe the nature and stability of the exposed soil. Design of excavation shoring systems should be performed by a qualified engineer.

For this project, we recommend that refusal, or "hard rock" be defined as materials that cannot be removed by a Cat D10 dozer with a single ripper or cannot be excavated with a Cat 345D excavator (or equivalent sized equipment) utilizing a 24-inch bucket equipped with rock teeth. Excavations that cannot be performed with larger excavation equipment will require blasting or use of a rock breaker to help facilitate excavation with heavy-duty excavators. Blasting should be performed in accordance with State and local regulations.

Surcharge loading by stockpiled soil, building materials and other loads should not be allowed directly adjacent to open trenches to prevent surcharge loading of the trench sidewalls. Materials and loads should be kept back from the edge of the trench at least half the trench depth or a distance of five feet, whichever is greater, unless the trench and any shoring used are appropriately designed for the anticipated induced load conditions. Vehicle and equipment traffic should be avoided near open trenches.

Groundwater Effect on Development

Groundwater, as discussed previously, was encountered at relatively shallow depths and is likely to vary with seasonal fluctuations in rainfall. The groundwater levels encountered in our test pits and borings are likely only typical of drier conditions.

We anticipate, based on historical imagery of surface ponding and observed surface features, that groundwater will experience seasonal fluctuations, with potential for development of surface ponding, seeps, and other surface manifestations of groundwater during wetter times of year (such as during winter months or after significant prolonged rainfall). Storm water and irrigation water will likely percolate through the near surface soils and perch on the relatively impermeable weathered and hard rock present beneath the sites.

Dewatering may be required for excavations at the site where seepage of water is encountered. The need for dewatering of excavations should be determined when subsurface conditions are fully exposed. We anticipate standard sump pit and pumping procedures should be adequate to control localized seepage encountered during construction if completed in the summer months. Trenching performed during the wetter portions of the year will likely require more extensive dewatering effort.

Near-surface soils will be in a near-saturated to saturated condition during and for a considerable period of time following the rainy season. Grading operations attempted following the onset of heavy precipitation and prior to prolonged drying periods will be hampered by high soil moisture contents. Such soils, intended for use as imported fill, will require considerable aeration to reach a moisture content that will permit the recommended compaction to be achieved. This should be considered in the construction schedule.

Existing swales and seasonal ponds will tend to collect and convey water even after site grading as percolating storm or irrigation water will flow along the preexisting paths. Lots currently crossed by seasonal swales, such as Lot 8, will require interceptor drains prior to placement of fill to control the flow of subsurface water under the lots.

Pavement Subgrade Quality

The near-surface soils anticipated to be encountered at pavement subgrade level are considered moderate quality materials for support of asphalt concrete pavements. Previous laboratory testing by Soil Search Engineering (SSE) of near-surface soils indicated that the surface material possesses a Resistance ("R") value of 37. Additional testing, completed in accordance with California Test 301 during the current study, produced an R-value of 69. We note that data from four R-value tests completed in similar soil conditions at a site approximately two miles south of the project site (WKA #10958.02) also yielded higher R-values of between 76 and 80.

Variation in the natural soil quality, along with variations in sampling procedure, can partially account for the variation seen in the R-values obtained from the silty sand. Based on this we anticipate that with uniform processing and compaction of the soils the higher R-values will be more representative of the overall in-situ performance. As such, based on the anticipated natural variations in soils quality, and our experience in the area, we have selected an R-value of 50 for design of asphalt concrete pavements.

Soil Corrosion Potential

One sample of near-surface soil from TP3 was submitted to Sunland Analytical Lab for testing to determine pH, chloride and sulfate concentrations, and minimum resistivity to help evaluate the soils corrosion potential. The results of the corrosivity testing are summarized in Table 2 and copies of the analytical test reports are presented in Figures A4 through A5.

TABLE 2
SOIL CORROSIVITY TESTING

Analyte	Test Method	TP3 (0'-3')
pH	CA DOT 643 Modified*	5.44
Minimum Resistivity	CA DOT 643 Modified*	7,500 Ω-cm
Chloride	CA DOT 417	5.1 ppm
Sulfate	CA DOT 422	5.1 ppm
	ASTM D516	5.5 ppm

NOTES:
* = Small cell method
Ω-cm = Ohm-centimeters
ppm = Parts per million

The California Department of Transportation Corrosion and Structural Concrete Field Investigation Branch, 2012 Corrosion Guidelines, considers a site to be corrosive to foundation elements if one or more of the following conditions exists for the representative soil and/or water samples taken: has a chloride concentration greater than or equal to 500 ppm, sulfate concentration greater than or equal to 2000 ppm, or a pH of 5.5 or less.

Based on this criterion, soils tested with a lower pH and may be considered corrosive to steel reinforcement properly embedded within Portland cement concrete (PCC).

Table 19.3.1.1 – Exposure Categories and Classes, of American Concrete Institute (ACI) 318-14, Section 19.3 – Concrete Design and Durability Requirements, as referenced in Section 1904.1 of the 2013 CBC, indicates the severity of sulfate exposure for the sample tested is Exposure Class S0 (water-soluble sulfate concentration in contact with concrete is low and

injurious sulfate attack is not a concern). However, additional requirements for concrete strength and design may be required for soils with low pH. The project structural engineer should evaluate the requirements of ACI 318-14 and determine their applicability to the site.

Wallace-Kuhl & Associates are not corrosion engineers. Therefore, if it is desired to further define the soil corrosion potential at the site a corrosion engineer should be consulted.

RECOMMENDATIONS

General

The recommendations presented below are appropriate for typical construction in the late spring through fall months. The on-site soils likely will be saturated by rainfall in the winter and early spring months, and will not be compactable without drying by aeration. Also, this site will be subject to perched groundwater at shallow elevations, that can cause instability in subgrade soils due to high moisture contents. Should the construction schedule require work to begin before the soils dry or to continue during the wet months, additional recommendations can be provided, as conditions dictate.

Site Clearing

Prior to site grading, the site should be cleared of all surface and subsurface structures associated with current and previous development of the site, including foundations, oversized rock (greater than about 3 inches in maximum dimension), rubbish, rubble, and deleterious debris, to expose firm and stable soils. Particular attention should be paid to the area of the former homestead and associated structures. Trees or shrubs designated for removal should include the entire rootball and all roots larger than ½-inch in diameter. Structures designated for removal should include the foundations and any associated utilities including the trench backfill. Adequate removal of debris and roots may require laborers handpicking to clear the subgrade soils to the satisfaction of the Geotechnical Engineer's representative. All debris should be removed from the site.

On-site wells and septic systems, if present, should be abandoned in accordance with Placer County Environmental Health Department requirements.

Existing surface vegetation and organically laden soil within construction areas should be removed by stripping. Strippings should not be used in fill construction in areas supporting structural improvements, pavements, or interior/exterior concrete slabs. Strippings may be stockpiled for later use in landscape areas or disposed of off-site. If used in landscape areas, the strippings should be kept at least five feet from the building pads and other surface improvements, moisture conditioned and compacted, with a maximum thickness that should not exceed a total depth of two feet. Discing of the organics into the surface soils may be a suitable alternate to stripping, depending on the condition and quantity of the organics at the time of grading. The decision to utilize discing in lieu of stripping should be approved by the Geotechnical Engineer *prior to use at the site*. Discing operations, if approved, should be observed by the Geotechnical Engineer's representative and be continuous until the organics are adequately mixed into the surface soils to provide a compactable mixture of soil containing minor amounts of organic matter. Pockets or concentrations of organics will not be allowed. *Under circumstances, discing of vegetation greater than 8 inches in height shall not be allowed.*

Depressions resulting from site clearing operations, as well as any loose, soft, disturbed, saturated, or organically contaminated soils, as identified by the Geotechnical Engineer's representative, should be cleaned out to firm, undisturbed soils and backfilled with imported fill in accordance with the recommendations of this report.

Subgrade Preparation

Following site clearing and organic removal, areas designated to receive fill or remain at-grade, should ripped and cross-ripped to a depth of at least 12 inches, thoroughly moisture conditioned to within two percent of the optimum moisture content, and uniformly compacted to at least 90 percent of the maximum dry density as determined by ASTM D1557. The intent of this recommendation is to expose and remove any remaining buried structures associated with previous or existing development, boulders, roots, or debris; and to provide adequate uniform compaction of the subgrade. All oversized rock fragments exposed should be removed or broken down into fragments that are no greater than 3 inches in maximum dimension.

Weathered and unweathered rock may not require ripping and recompaction if exposed during site preparation and excavation. The Geotechnical Engineer should determine whether scarification and compaction is required based on the exposed conditions.

Engineered Fill Construction

On-site soil and rock materials primarily less than 3 inches in maximum dimension may be used as engineered fill provided they are processed to a uniform compactable consistency. Screening and processing of on-site materials may be necessary to achieve primarily 3-inch minus material. Large rocks that cannot be uniformly incorporated into the engineered fill should be broken down into smaller pieces less than 3 inches in maximum dimension or removed from the fill.

Imported fill, if necessary, should be compactable, well-graded, coarse grained soils (as defined by ASTM D2487) with a Plasticity Index of 15 or less when tested in accordance with ASTM D4318; an Expansion Index of 20 or less when tested in accordance with ASTM D4829, and should not contain particles greater than 3 inches in maximum dimension. Import fill materials to be used in pavement areas should have a Resistance ("R") value greater than 50 when tested in accordance with California Test 301. In addition, we recommend that the contractor supply a certification for any imported fill materials, other than aggregate base, that indicates the fill materials are free of known contaminants, and have satisfactory corrosion characteristics. Imported soils should be approved by the Geotechnical Engineer prior to being transported to the site.

Engineered fill materials should be moisture conditioned to at least the optimum moisture content, and should be compacted to at least 90 percent of the maximum dry density per ASTM D 1557. In place, compacted, density should be confirmed by Geotechnical Engineer or their representative using conventional field density testing procedures. In order to achieve adequate compaction throughout each lift, compacted lift thicknesses should not exceed 6 inches and nesting of gravels and cobbles should be avoided. Typically, placement of soils in loose lifts not exceeding 8 inches will result in acceptable lift thicknesses.

Compactive effort should be applied uniformly across the full width of fill construction using appropriate equipment. Appropriate equipment for predominantly coarse grained cohesionless soils or soils containing less than 20 percent fines typically includes vibratory rollers, vibratory plates, or other forms of vibratory compaction equipment. Cohesive soils containing more than 20 percent fines with some cohesion may be more appropriately compacted using a sheep's-foot roller, soil wheel, or similar compaction equipment. Power tampers or rammers typically may be used with varying amounts of effort in areas with restricted access. We anticipate that after processing the on-site engineered fill will be coarse grained silty sands (SM). Silty sands are typically easy to moderately difficult to compact, potentially requiring special attention to moisture content and multiple passes of some form of vibratory compaction equipment such as vibratory rollers or power tampers/rammers.

To reduce the potential for differential settlement of building foundations, all fill placed in lots developed with more than five feet of fill should be compacted to at least 95 of the maximum dry unit weight.

Where building pads will be created by excavation and fill, partially exposing weathered rock, the building pad, including the area five feet beyond the outside edge of the perimeter foundation, may either be uniformly overexcavated to a depth at least two feet below finished soil pad grade and restored with engineered fill, or the foundations for the structures may be deepened so that all foundations bear on either engineered fill or weathered rock. The project grading plans and foundation plans should clearly indicate this recommendation on the affected lots. The Geotechnical Engineer should review the final grading plans and observe the contractor during construction to verify that the areas and extent of potential over-excavation have been completed.

The upper six inches of final pavement subgrades should be properly processed, thoroughly moisture conditioned to within two percent of the optimum moisture content and uniformly compacted to at least 95 percent of the maximum dry density as determined by ASTM D1557, regardless of whether final grade is achieved by excavation, imported fill, or left at-grade. Compaction of pavement subgrades should be accomplished just prior to placement of aggregate base materials. Where weathered bedrock is exposed at the pavement subgrade elevation, the requirement to scarify and compact the subgrade may be waived if approved by the Geotechnical Engineer.

Fills constructed on sloping ground in excess of a five percent slope should be made such that adequate compaction is achieved throughout. Soils should be placed in level, uniform lifts and should be keyed into existing slopes a minimum distance of one foot. Compacted soils should also be keyed into the toe of the slope one-foot vertical, with the subgrade of the keyway compacted as described above previously for preparation of subgrades. If final grading includes a slope, soils should be overbuilt, compacted, and then cutback to achieve the finished grade.

All earthwork operations should be accomplished in accordance with the recommendations contained within this report and the attached earthwork specifications. We recommend that the representative of the Geotechnical Engineer retained by the owner during construction be present during site preparation and fill placement to verify compliance with these recommendations and the project specifications. Periodic classification and testing of compacted fill materials is required per section 1705.6 of the 2013 CBC. In addition, continuous observation of materials, densities, and lift thicknesses during placement and compaction of fill is also required.

<u>Subdrains</u>

The grading plans for the project indicate all or part of the drainage swales located on the northern and southern ends of the site may be filled as part of site development, and that a subdrain may be needed to control subsurface drainage conditions that will be created within the filled swales. In addition, subdrains may be required as part of an overall water management plan to limit development of seeps or surface ponding. Subdrains should be designed to adequately carry water collected in the system, and should have a designated drainage outlet at an appropriate location.

We recommend that at a minimum subdrains be placed in the flow line of the swale beneath Lot 8. Subdrains in existing swales to be filled should be constructed by excavation of a trench at least 18 inches wide and at least one-foot-deep, sloped to discharge at no less than one percent fall. A drainpipe consisting of six-inch diameter, perforated Schedule 40 PVC should be placed on an approximately four-inch layer of State of California Class 2 permeable material (Caltrans Specification 68-2.02F(3)), and then covered to the top of the trench with the Class 2 permeable material. Open-graded crushed gravel may be used in lieu of the Class 2 permeable rock, provided that the gravel and pipe are completely enveloped in an approved non-woven geotextile filter fabric.

The location of the drains should be documented on the as-built grading plans and provided to future owners of the affected lots so that the drains are not disturbed by construction of improvements such as swimming pools, spas, or other excavations in the vicinity of the drains.

We recommend that the final subdrain design be reviewed and approved by the Geotechnical Engineer prior to use on the site.

<u>Utility Construction and Trench Backfill</u>

Excavation conditions for underground utility construction are described in the *Excavation Conditions* section of this report. Seepage into utility excavations due to perched water may be encountered in isolated locations throughout the site. Deeper excavations, if needed, will require dewatering. The chances of encountering seepage or elevated groundwater levels will be greater during the winter and spring months of the year.

Initial backfill and embedment for utility construction (e.g. pipe zone backfill) should conform to the pipe manufacturer's recommendations and applicable governing agency standards. Intermediate backfill (e.g. trench zone) should extend from the top of the initial backfill to within 12 inches of the finished grade and should use imported or engineered fill. Intermediate backfill should be placed and compacted to 90 percent of the maximum dry density per ASTM D 1557 in 6 inch lifts. Jetting as the sole means of compaction is <u>not</u> recommended. The final zone backfill in the upper 12 inches should conform to recommendations for the subgrade or pavement recommendations, as appropriate.

Manholes should be founded on engineered fill compacted to a minimum of 95 percent relative compaction per ASTM D 1557 with a minimum thickness of 12 inches. If weathered or unweathered rock is encountered, 12 inches of compacted select or imported fill may not be required, as approved by the Geotechnical Engineer. Backfill around manholes shall conform to the applicable standards for pipe zone, trench zone, and final zone backfill discussed above.

Underground utility trenches that are aligned nearly parallel with foundations should be *at least* three feet from the outer edge of foundations, wherever possible. As a general rule, trenches should not encroach into the zone extending outward at a one horizontal to one vertical (1:1) inclination below the bottom of the foundations, and typically should not remain open longer than 72 hours. The intent of these recommendations is to prevent loss of both lateral and vertical support of foundations, resulting in possible settlement or failure of the foundations.

Trench backfill materials and compaction requirements for utility lines within the public right-of-way should conform to applicable city and/or county requirements.

Foundation Design

The proposed structures may be supported upon continuous and/or isolated spread foundations based entirely within: 1) undisturbed native surface soils, imported fill, weathered rock or a combination of these materials; or 2) undisturbed weathered or hard rock, as determined by the Geotechnical Engineer or their representative. It is emphasized that the structure should not be supported partially upon rock and partially upon undisturbed native soils or imported fill materials. Some deepening of the foundation excavations may be required to reach the appropriate bearing materials, as determined by the Geotechnical Engineer or their representative. We recommend bid documents to include a unit price per foot of additional foundation excavation, as needed.

Continuous foundations should be at least 12 inches wide and isolated spread foundations should maintain a minimum 24-inch width dimension in either direction. One- and two-story structure foundations should extend at least 12 inches below building pad soil subgrade. For this project, the building pad subgrade shall be defined as the surface upon which capillary break gravel is placed or any surrounding compacted soil grade, whichever is lower.

Foundations so established may be sized for an allowable soil bearing pressure of 2500 pounds per square foot (psf) for dead plus live loads. A 1/3 increase of the bearing capacity may be used for foundation designs that include the short-term loading effects of wind and/or seismic forces. The weight of the foundation concrete extending below lowest adjacent soil grade may be disregarded in sizing computations. Foundation size and reinforcement should be determined by the project structural engineer.

Resistance to lateral foundation displacement may be computed using an allowable friction factor of 0.35, which may be multiplied by the effective vertical load on each foundation. Additional lateral resistance may be computed using an allowable passive earth pressure of 400 psf per foot of depth. These two modes of resistance should not be added unless the frictional value is reduced by 50 percent since full mobilization of these resistances typically occurs at different degrees of horizontal movement.

We recommend that all foundation excavations be observed by the Geotechnical Engineer prior to placement of reinforcement and concrete to verify suitable bearing materials and conditions are exposed. Periodic inspection is required per Section 1705.6 of the 2013 CBC.

Interior Floor Slab Support

Interior concrete slab-on-grade floors should be at least four inches thick and can be supported upon the soil subgrade prepared in accordance with the recommendations in this report. We recommend that interior floor slabs be reinforced to provide structural continuity, mitigate cracking and permit spanning of local soil irregularities. The project design engineer should determine final floor slab reinforcing requirements. Concrete curing and joint spacing and details should conform to current Portland Cement Association (PCA) and ACI guidelines.

Floor slabs may be underlain by a layer of free-draining crushed rock, serving as a deterrent to migration of capillary moisture. The crushed rock layer should be at least four inches and no more than six inches thick and should be graded such that 100 percent passes a one-inch sieve and less than five percent passes a No. 4 sieve. Additional moisture protection may be provided by placing a vapor retarder membrane (at least 10-mils thick) directly over the crushed rock. The membrane should meet or exceed the minimum specifications as outlined in ASTM E1745, and be installed in strict conformance with the manufacturer's recommendations.

Floor slab construction over the past 30 years or more has included placement of a thin layer of sand over the vapor retarder membrane. The intent of the sand is to aid in the proper curing of the slab concrete. However, recent debate over excessive moisture vapor emissions from floor slabs includes concern for water trapped within the sand. As a consequence, we consider the use of the sand layer as optional. The concrete curing benefits should be weighed against efforts to reduce slab moisture vapor transmission.

The recommendations presented above are intended to reduce significant soils-related cracking of the slab-on-grade floors. More important to the performance and appearance of a Portland cement concrete slab is the quality of the concrete, the workmanship of the concrete contractor, the curing techniques utilized, and the spacing of control joints.

Floor Slab Moisture Penetration Resistance

It is considered likely that interior floor slab subgrade soils will become wet to near-saturated at some time during the life of the structures. This is a certainty when slabs are constructed during the wet season or when constantly wet ground or poor drainage conditions exist adjacent to structures. For this reason, it should be assumed that all interior slabs in occupied areas, as well as those intended for moisture-sensitive floor coverings or materials, require protection against moisture or moisture vapor penetration. Standard practice includes the crushed rock and water vapor retarder as suggested above. However, the gravel and membrane offer only a limited, first-line of defense against soil-related moisture. Recommendations contained in this report concerning foundation and floor slab design are presented as *minimum* requirements, only from the geotechnical engineering standpoint.

It is emphasized that the use of sub-slab crushed rock and vapor retarder membrane will not "moisture proof" the slab, nor does it assure that slab moisture transmission levels will be low enough to prevent damage to floor coverings or other building components. If increased protection against moisture vapor penetration of slabs is desired, a concrete moisture protection specialist should be consulted. The design team should consider all available measures for slab moisture protection. It is commonly accepted that maintaining the lowest practical water-cement ratio in the slab concrete is one of the most effective ways to reduce future moisture vapor penetration of the completed slabs.

Exterior Flatwork (Non-Pavement Areas)

Areas to receive exterior concrete flatwork (e.g., sidewalks) should be uniformly moisture conditioned to within two percent of the optimum moisture content, and compacted to at least 90 percent relative compaction based on ASTM D1557.

Proper moisture conditioning of the subgrade soils is essential to the performance of exterior flatwork. Uniform moisture conditioning of subgrade soils is important to reduce the risk of non-uniform moisture withdrawal from the concrete and the possibility of plastic shrinkage cracks. Practices recommended by the PCA for proper placement and curing of concrete should be followed during exterior concrete flatwork construction.

We recommend the concrete flatwork be constructed with thickened edges in accordance with ACI design standards, latest edition. Flatwork should be at least four inches thick and reinforced for crack control, if necessary. The flatwork reinforcement should be provided by the civil engineer or project architect. Accurate and consistent location of the reinforcement at mid-slab is essential to its performance. The slab designer should determine if exterior flatwork should be constructed independent of (or connected to) the building foundations.

Retaining Walls

The vesting tentative map prepared by Meredith Engineering (dated August 9, 2016) indicates a retaining wall along the southerly property line, and along portions of the west property line of the planned subdivision. The retaining walls are indicated to be 1 to 7.5 feet in height. A portion of the west property line also will be graded to slope from west to east at a three horizontal to one vertical (3H:1V) inclination.

Retaining walls that are allowed to yield or rotate at the top should be capable of resisting "active" lateral soil pressures equal to an equivalent fluid pressure of 30 psf per foot of retained soil. Rigid or restrained retaining walls that are not allowed to yield at the top should be capable of resisting "at-rest" lateral soil pressures equal to an equivalent fluid pressure of 50 psf per foot of retained soil. These soil pressures assume a horizontal grade behind the walls, that all soil retained by the walls will be onsite or imported granular soils, and that the walls will be fully drained so that hydrostatic pressures will not develop behind the walls.

Retaining walls may be subjected to surcharge loads produced by sloping backfills, nearby building foundations, vehicular traffic and parking, as well as construction equipment and material storage. These additional surcharge loads should be considered in the retaining wall design. Appropriate design parameters can be provided on a case-by-case basis, where required.

Retaining walls may be supported upon shallow foundations extending at least 12 inches below lowest adjacent site grade, or to rock, and may be designed using the applicable vertical and lateral bearing capacity recommendations contained in the *Foundation Design* section of this report.

Drainage behind retaining walls can be accomplished using 12-inch wide gravel drainage layer behind the walls that extends from the bottom of the wall to within 12 inches of the top of the wall. The top foot of soil above the drainage layer should consist of compacted on-site materials, unless covered by a slab or pavement. The gravel drain should consist of Class 2 permeable material as defined in the Caltrans Standard Specifications, latest edition, or ¾-inch crushed rock, wrapped in a nonwoven geotextile fabric such as Mirafi 140N or equivalent.

Weep holes or perforated drain pipes should be provided at the base of the retaining wall to collect and discharge accumulated water. Drain pipes, if used, should drain at a minimum one percent slope to an appropriate drainage system. Proprietary geotextile composites, such as Miradrain 6200 or equivalent, may be used in lieu of gravel drainage, if approved by the Geotechnical Engineer.

The soils behind the wall, including the drainage layer, will be moist to saturated at various times of the year. Where moisture penetration or efflorescence of the face of retaining walls is undesirable, the back of the retaining wall should be water proofed, or other means implemented to prevent infiltration of moisture through the walls.

Approved on-site or imported granular free draining soils should be used to backfill retaining walls. Backfill should be placed in lifts not exceeding 6 inches in compacted thickness and compacted by mechanical methods to at least 90 percent of the ASTM D1557 maximum dry density.

Surface Drainage

Surface drainage should be accomplished to provide positive drainage of surface water away from the homes and other structures and drain into a nearby drainage collection system. The subgrade adjacent to the buildings should be sloped away from foundations at a minimum of at least a two percent gradient for a minimum of 10 feet, where possible. Roof gutter downspouts and surface drains should drain onto pavements or be connected to rigid, non-perforated piping directed to an appropriate drainage point away from the structures. Ponding of surface water should not be allowed adjacent to the buildings or pavements. Landscape berms, if planned, should not be constructed in such a manner as to promote drainage toward the buildings.

Pavement Design

Specific pavement design standards for the City of Loomis were not available to us at the time this report was prepared. The following pavement sections are applicable for on-site and off-site roads.

We have assumed typical traffic indices of 4.5, 6.0, and 7.0 for private pavements and traffic indices of 6.0, 7.0, 8.0, 9.0, and 10.0 for public pavements. The project civil engineer should select the appropriate traffic index based on anticipated traffic conditions, and any applicable City or County requirements. We can provide additional pavement section alternatives based on alternate traffic indices, upon request. The following tables of pavement sections have been calculated based on the assumed traffic indices using an R-value of 50, and the procedures contained within the 6th Edition of the *California Highway Design Manual.*

TABLE 3
ON-SITE PAVEMENT DESIGN ALTERNATIVES
R-VALUE = 50

Traffic Index (TI)	Type B Asphalt Concrete (inches)	Class 2 Aggregate Base (inches)	Portland Cement Concrete (inches)
4.5	2½	4	--
	--	4	4
6.0	2½	6	--
	3½*	4	--
	--	4	4
7.0	3	7	--
	4*	5	--
	--	5	4

*Asphalt thickness includes Caltrans Factor of Safety.

TABLE 4
OFF SITE PAVEMENT DESIGN ALTERNATIVES
R-VALUE = 50

Traffic Index (TI)	Type B Asphalt Concrete (inches)	Class 2 Aggregate Base (inches)	Portland Cement Concrete (inches)
6.0	2½	6	--
	3½*	4	--
	--	4	4
7.0	3	7	--
	4*	5	--
	--	5	4
8.0	4	7	--
	5*	6	--
	--	6	5
9.0	4	9	--
	5½*	7	--
	--	7	5
10.0	5	10	--
	6½*	8	--
	--	8	6

* Asphalt thickness includes Caltrans Factor of Safety.

We emphasize that the performance of pavements is critically dependent upon uniform compaction of the subgrade soils, as well as all imported fill and utility trench backfill within the limits of the pavements. The Class 2 aggregate base (Class 2 AB) should conform to the Caltrans standard specifications, latest edition, and should be compacted to at least 95 percent maximum dry density per ASTM D1557. Final subgrade preparation should be performed just prior to placement of the aggregate base.

High axle loads coupled with shear stresses induced by sharply turning tire movements can lead to failure in asphalt concrete pavements. Therefore, we recommend that consideration be given to using an appropriate PCC section in areas subjected to concentrated heavy wheel loading, such as entry driveways and in front of trash enclosures.

We recommend PCC slabs be constructed with thickened edges in accordance with ACI design standards, latest edition. Reinforcing for crack control, if desired, should be determined by the project civil engineer. Joint spacing and details should conform to current PCA or ACI guidelines. Portland cement concrete should achieve a minimum compressive strength of 3500 pounds per square inch at 28 days.

Efficient drainage of all surface water to avoid infiltration and saturation of the supporting aggregate base and subgrade soils is important to pavement performance. Materials quality and construction of the structural section should conform to the applicable provisions of the Caltrans Standard Specifications, latest edition, and applicable city and county requirements as applicable.

<u>Geotechnical Engineering Observation and Testing During Construction</u>

Site preparation should be accomplished in accordance with the recommendations of this report and the *Earthwork Specifications* provided in Appendix B. Geotechnical testing and observation during construction is considered a continuation of the geotechnical engineering investigation. Wallace-Kuhl & Associates should be retained to provide testing and observation services during site earthwork and foundation construction to verify compliance with this geotechnical report and the project plans and specifications, and to provide consultation as required during construction. These services are beyond the scope of work authorized for this study.

Many factors can affect the number of tests that should be performed during the course of construction, such as soil type, soil moisture, season of the year and contractor operations/performance. Therefore, it is crucial that the actual number and frequency of testing be determined by the Geotechnical Engineer or their representative during construction based on their observations, site conditions, and difficulties encountered.

In the event that Wallace-Kuhl & Associates is not retained to provide geotechnical engineering observation and testing services during construction, the Geotechnical Engineer retained to provide these services should indicate in writing that they agree with the recommendations of this report, or prepare supplemental recommendations as necessary. A final report by the "Geotechnical Engineer" should be prepared upon completion of the project.

LIMITATIONS

Our recommendations are based upon the information provided regarding the proposed construction, combined with our analysis of site conditions revealed by the field exploration and laboratory testing programs. We have used prudent engineering judgment based upon the information provided and the data generated from current and previous investigations. This report has been prepared in substantial compliance with generally accepted geotechnical engineering practices that exist in the area of the project at the time the report was prepared. No warranty, either express nor implied, is provided.

If the proposed construction is modified or relocated or, if it is found during construction that subsurface conditions differ from those we encountered at the test pit locations, we should be afforded the opportunity to review the new information or changed conditions to determine if our conclusions and recommendations must be modified.

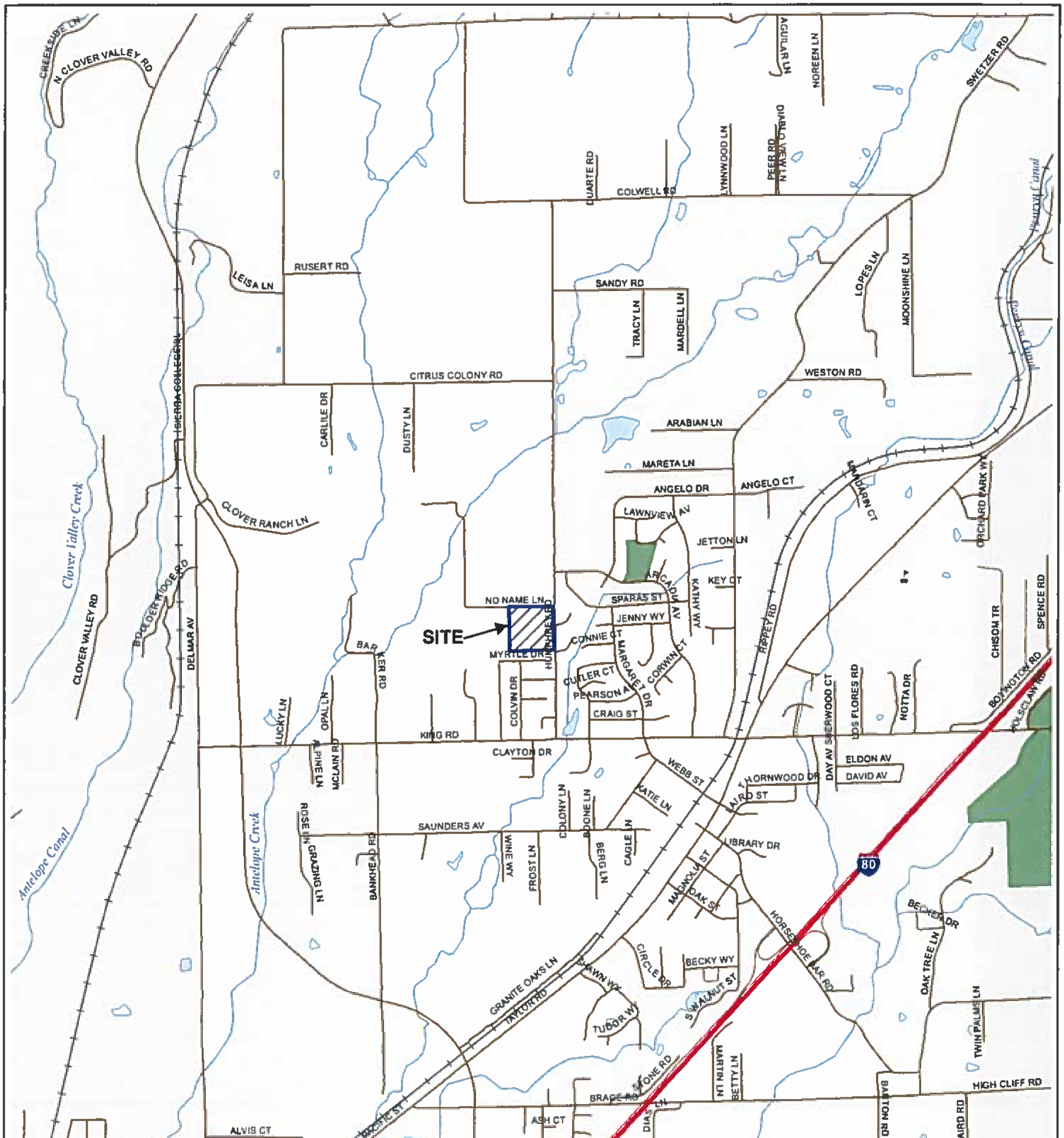
We emphasize that this report is applicable only to the proposed construction and the investigated site. This report should not be utilized for construction on any other site. This report is considered valid for the proposed construction for a period of two years following the date of this report. If construction has not started within two years, we must re-evaluate the recommendations of this report and update the report, if necessary.

Wallace - Kuhl & Associates

REGISTERED PROFESSIONAL ENGINEER
DAVID R. GIUS, JR.
NO. 2318
Exp. 9/30/16
GEOTECHNICAL
STATE OF CALIFORNIA

Alexander D. Wright
Staff Engineer

David R. Gius, Jr., GE
Senior Engineer



Street data courtesy of Placer County.
Hydrography courtesy of the U.S. Geological Survey
acquired from the GIS Data Depot, December, 2007.
Projection: NAD 83, California State Plane, Zone II

N

0 1,000 2,000
Feet

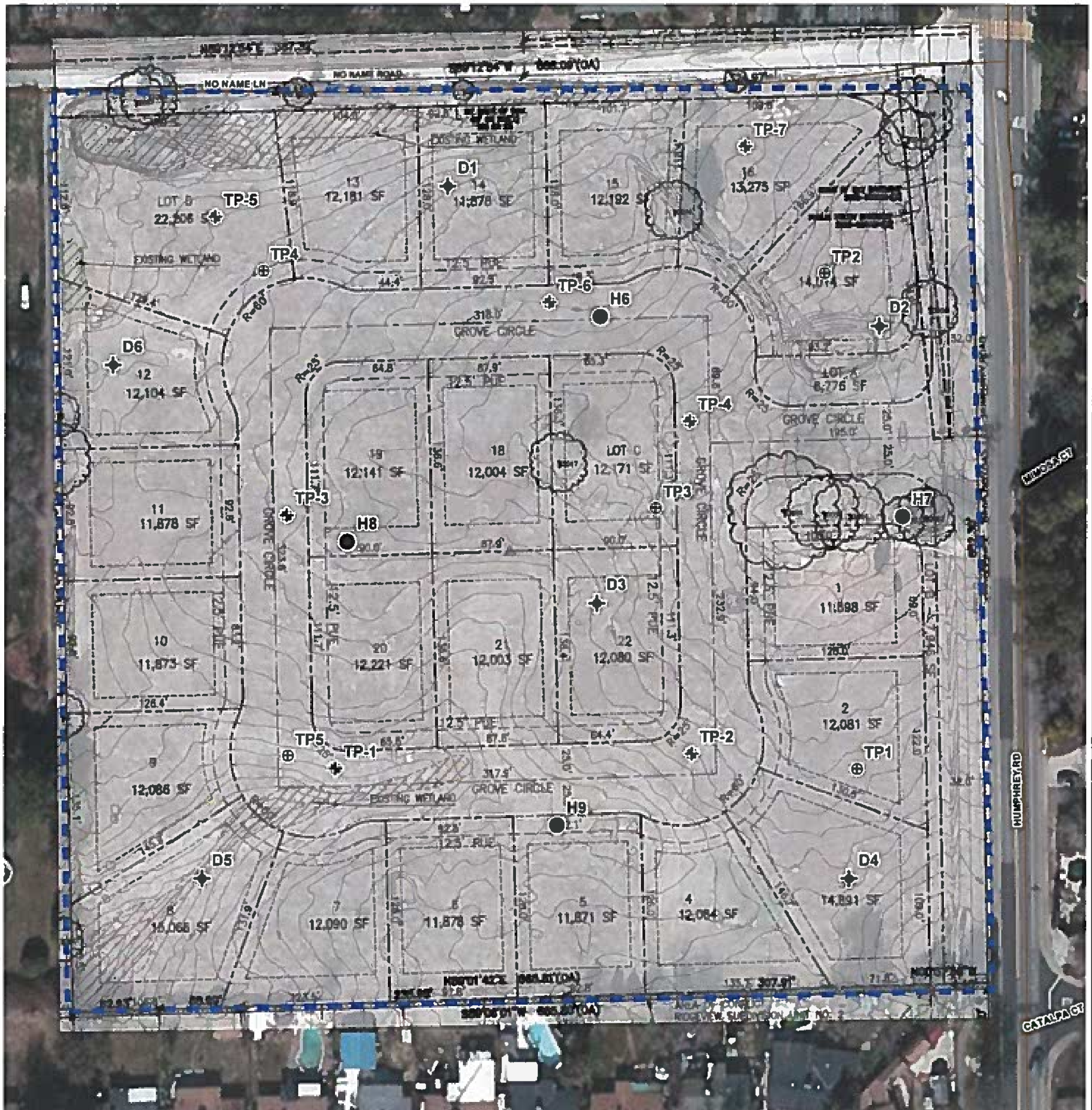
WallaceKuhl & ASSOCIATES

VICINITY MAP

THE GROVE

Loomis, California

FIGURE	1
DRAWN BY	RWO
CHECKED BY	JRY
PROJECT MGR	DRG
DATE	08/16
WKA NO.	11071.01



Aerial provided by ESRI.
Site Plan adapted from the Proposed Lotting Exhibit provided by Meredith Engineering, dated August 3, 2016.
Projection: NAD 83, California State Plane, Zone II

Legend

- Site Boundary
- Approximate Soil Boring Location
- Approximate Test Pit Location
- Approximate Soil Boring Location (SSE, 2005)
- Proposed Test Pit Location (SSE, 2005)

N

0 50 100
Feet

WallaceKuhl & ASSOCIATES

SITE PLAN

THE GROVE

Loomis, California

FIGURE	2
DRAWN BY	RWO
CHECKED BY	JRY
PROJECT MGR	DRG
DATE	08/16
WKA NO.	11071.01

Project: The Grove Project Location: Sacramento, California WKA Number: 11071.01	LOG OF SOIL BORING D1 Sheet 1 of 1

Date(s) Drilled	8/1/16	Logged By	JRY	Checked By	MMW
Drilling Method	Solid Stem Auger	Drilling Contractor	V&W Drilling, Inc.	Total Depth of Drill Hole	12.0 feet
Drill Rig Type	CME 75	Diameter(s) of Hole, inches	6"	Approx. Surface Elevation, ft MSL	
Groundwater Depth [Elevation], feet	8.0	Sampling Method(s)	Modified California	Drill Hole Backfill	Soil cuttings
Remarks				Driving Method and Drop	140-lb automatic hammer, 30 inch drop

ELEVATION, feet	DEPTH, feet	GRAPHIC LOG	ENGINEERING CLASSIFICATION AND DESCRIPTION	SAMPLE DATA: SAMPLE	SAMPLE DATA: SAMPLE NUMBER	SAMPLE DATA: NUMBER OF BLOWS	TEST DATA: MOISTURE CONTENT, %	TEST DATA: DRY UNIT WEIGHT, pcf	TEST DATA: ADDITIONAL TESTS
			Light brown/brown, slightly moist, medium dense, silty fine to coarse SAND (SM) with intensely weathered, granodioritic rock		D1-1I	20			
	5		Brown, moist, very dense, intensely WEATHERED, granodioritic ROCK (RX) with sand.		D1-2I	73			
	10		Wet, with trace clay		D1-3I	50/6"			
			Refusal at 12 feet below grade Groundwater encountered at 8 feet						

BORING LOG 11071.01 - THE GROVE.GPJ WKA.GDT 8/26/16 8:30 AM

Wallace Kuhl & ASSOCIATES

FIGURE 3

Project: The Grove
Project Location: Sacramento, California
WKA Number: 11071.01

LOG OF SOIL BORING D2

Date(s) Drilled	8/1/16	Logged By	JRY	Checked By	MMW
Drilling Method	Solid Stem Auger	Drilling Contractor	V&W Drilling, Inc.	Total Depth of Drill Hole	9.5 feet
Drill Rig Type	CME 75	Diameter(s) of Hole, inches	6"	Approx. Surface Elevation, ft MSL	
Groundwater Depth [Elevation], feet	NA	Sampling Method(s)	Modified California	Drill Hole Backfill	Neat cement
Remarks				Driving Method and Drop	140-lb automatic hammer, 30 inch drop

ELEVATION, feet	DEPTH, feet	GRAPHIC LOG	ENGINEERING CLASSIFICATION AND DESCRIPTION	SAMPLE DATA: SAMPLE	SAMPLE DATA: SAMPLE NUMBER	SAMPLE DATA: NUMBER OF BLOWS	TEST DATA: MOISTURE CONTENT, %	TEST DATA: DRY UNIT WEIGHT, pcf	TEST DATA: ADDITIONAL TESTS
			Light brown/brown, slightly moist, medium dense, silty fine to coarse SAND (SM) with intensely weathered, granodioritic rock		D2-1I	17	6.4	107	
			Dense		D2-2I	40	6.1	118	
	5								
			Grayish brown, moist, very dense, intensely WEATHERED, granodioritic ROCK (RX) with sand		D2-3I	50/6"			
			Refusal at 9.5 feet below grade Groundwater not encountered						

Wallace Kuhl & ASSOCIATES

FIGURE 4

Project: The Grove Project Location: Sacramento, California WKA Number: 11071.01	LOG OF SOIL BORING D3 Sheet 1 of 1

Date(s) Drilled	8/1/16	Logged By	JRY	Checked By	MMW
Drilling Method	Solid Stem Auger	Drilling Contractor	V&W Drilling, Inc.	Total Depth of Drill Hole	12.0 feet
Drill Rig Type	CME 75	Diameter(s) of Hole, inches	6"	Approx. Surface Elevation, ft MSL	
Groundwater Depth [Elevation], feet	11.0	Sampling Method(s)	Modified California	Drill Hole Backfill	Soil cuttings
Remarks				Driving Method and Drop	140-lb automatic hammer, 30 inch drop

ELEVATION, feet	DEPTH, feet	GRAPHIC LOG	ENGINEERING CLASSIFICATION AND DESCRIPTION	SAMPLE (SAMPLE DATA)	SAMPLE NUMBER	NUMBER OF BLOWS	MOISTURE CONTENT, % (TEST DATA)	DRY UNIT WEIGHT, pcf	ADDITIONAL TESTS
			Light brown, slightly moist, dense, silty fine to coarse SAND (SM) with intensely weathered, granodioritic rock		D3-1I	49			TR
	5		Light brown, moist, dense, intensely WEATHERED, granodioritic ROCK (RX) with sand.		D3-2I	50			
	10		Wet, very dense		D3-3I	50/6"			

Refusal at 12 feet below grade
Groundwater encountered at 11 feet

BORING LOG 11071.01 - THE GROVE.GPJ WKA.GDT 8/26/16 8:30 AM

Wallace Kuhl & Associates

Project: The Grove
Project Location: Sacramento, California
WKA Number: 11071.01

LOG OF SOIL BORING D4

Date(s) Drilled	8/1/16	Logged By	JRY	Checked By	MMW
Drilling Method	Solid Stem Auger	Drilling Contractor	V&W Drilling, Inc.	Total Depth of Drill Hole	14.5 feet
Drill Rig Type	CME 75	Diameter(s) of Hole, inches	6"	Approx. Surface Elevation, ft MSL	
Groundwater Depth [Elevation], feet	13.0	Sampling Method(s)	Modified California	Drill Hole Backfill	Soil cuttings
Remarks				Driving Method and Drop	140-lb automatic hammer, 30 inch drop

ELEVATION, feet	DEPTH, feet	GRAPHIC LOG	ENGINEERING CLASSIFICATION AND DESCRIPTION	SAMPLE	SAMPLE NUMBER (SAMPLE DATA)	NUMBER OF BLOWS (SAMPLE DATA)	MOISTURE CONTENT, % (TEST DATA)	DRY UNIT WEIGHT, pcf (TEST DATA)	ADDITIONAL TESTS (TEST DATA)
			Grayish light brown, slightly moist, medium dense, silty fine to coarse SAND (SM) with intensely weathered, granodioritic rock		D4-1I	17			
			Grayish brown, moist, dense, intensely WEATHERED, granodioritic ROCK (RX) with sand.		D4-2I	49	6.3	127	
	5								
			Very dense		D4-3I	50/6"			
	10								
			Wet		D4-4I	50/6"			
			Refusal at 14.5 feet below grade Groundwater encountered at 13 feet						

BORING LOG 11071.01 - THE GROVE.GPJ WKA.GDT 8/26/16 8:30 AM

WallaceKuhl & ASSOCIATES

FIGURE 6

Project: The Grove Project Location: Sacramento, California WKA Number: 11071.01	**LOG OF SOIL BORING D5** Sheet 1 of 1

Date(s) Drilled	8/1/16	Logged By	JRY	Checked By	MMW
Drilling Method	Solid Stem Auger	Drilling Contractor	V&W Drilling, Inc.	Total Depth of Drill Hole	12.5 feet
Drill Rig Type	CME 75	Diameter(s) of Hole, inches	6"	Approx. Surface Elevation, ft MSL	
Groundwater Depth [Elevation], feet	NA	Sampling Method(s)	Modified California	Drill Hole Backfill	Soil cuttings
Remarks				Driving Method and Drop	140-lb automatic hammer, 30 inch drop

ELEVATION, feet	DEPTH, feet	GRAPHIC LOG	ENGINEERING CLASSIFICATION AND DESCRIPTION	SAMPLE DATA: SAMPLE	SAMPLE NUMBER	NUMBER OF BLOWS	TEST DATA: MOISTURE CONTENT, %	DRY UNIT WEIGHT, pcf	ADDITIONAL TESTS
			Brown, moist, medium dense, silty fine to coarse SAND (SM) with intensely weathered, granodioritic rock		D5-1I	14	13.0	112	
	5		Grayish brown, moist, very dense, intensely WEATHERED, granodioritic ROCK (RX) with sand.		D5-2I	50/6"			
	10				D5-3I	50/6"			
			Refusal at 12.5 feet below grade Groundwater not encountered						

BORING LOG 11071.01 - THE GROVE.GPJ WKA.GDT 8/26/16 8:30 AM

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FIGURE 7

Project: The Grove
Project Location: Sacramento, California
WKA Number: 11071.01

LOG OF SOIL BORING D6

Date(s) Drilled	8/1/16	Logged By	JRY	Checked By	MMW
Drilling Method	Solid Stem Auger	Drilling Contractor	V&W Drilling, Inc.	Total Depth of Drill Hole	9.5 feet
Drill Rig Type	CME 75	Diameter(s) of Hole, inches	6"	Approx. Surface Elevation, ft MSL	
Groundwater Depth [Elevation], feet	NA	Sampling Method(s)	Modified California	Drill Hole Backfill	Soil cuttings
Remarks				Driving Method and Drop	140-lb automatic hammer, 30 inch drop

ELEVATION, feet	DEPTH, feet	GRAPHIC LOG	ENGINEERING CLASSIFICATION AND DESCRIPTION	SAMPLE DATA: SAMPLE	SAMPLE DATA: SAMPLE NUMBER	SAMPLE DATA: NUMBER OF BLOWS	TEST DATA: MOISTURE CONTENT, %	TEST DATA: DRY UNIT WEIGHT, pcf	TEST DATA: ADDITIONAL TESTS
			Grayish brown, slightly moist, medium dense, silty fine to coarse SAND (SM) with intensely weathered, granodioritic rock		D6-1I	29			
			Grayish brown, moist, dense, intensely WEATHERED, granodioritic ROCK (RX) with sand.		D6-2I	37			
	5								
					D6-3I	50/6"			

Refusal at 9.5 feet below grade
Groundwater not encountered

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FIGURE 8

LOGS OF TEST PITS
The Grove
Excavated on August 1st, 2016, with a Case 580 Super M excavator
Logged by: Alexander Wright
WKA No. 11071.01

TEST PIT 1

0' to 2'	Brown, dry, medium dense, silty fine to coarse SAND (SM) with trace amounts of fine gravel/granodioritic rock
2' to 5.5'	Reddish brown/brown, dry to moist, dense to very dense, intensely weathered to decomposed granodioritic ROCK (RX) with fine to coarse sand.

Bottom of test pit at 5½ feet below existing ground surface
Groundwater was not encountered
Note outcrop of massive unweathered granodioritic rock observed approx. 10 to 15 feet southwest of test pit

TEST PIT 2

0' to 2'	Brown, dry, medium dense, silty fine to coarse SAND (SM) with trace amounts of fine gravel/granodioritic rock
2' to 5'	Reddish brown/brown, dry to moist, dense to very dense, intensely weathered to decomposed granodioritic ROCK (RX) with fine to coarse sand.

Bottom of test pit at 5 feet below existing ground surface
Groundwater was not encountered

TEST PIT 3

0' to 2.5'	Dark reddish brown, dry, medium dense, silty fine to coarse SAND (SM)
2.5' to 4.8'	Reddish brown/brown, dry to moist, dense to very dense, intensely weathered to decomposed granodioritic ROCK (RX) with coarse sand.

Bottom of test pit at 4.8 feet below existing ground surface
Groundwater was not encountered
R Value and Gradation Test completed on bulk sample taken from 0 to 2 feet

TEST PIT 4

0' to 2'	Brown, medium dense, dry, silty fine to coarse SAND (SM)
2' to 4.5'	Reddish brown, dry to moist, dense to very dense, intensely weathered to decomposed granodioritic ROCK (RX) with coarse sand.
4.5' to 7.5'	Grades to yellowish brown

Bottom of test pit at 7.5 feet below existing ground surface
Groundwater was not encountered

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LOGS OF TEST PITS

THE GROVE

Plumas Lakes, California

FIGURE	9
DRAWN BY	RWO
CHECKED BY	ADW
PROJECT MGR	DRG
DATE	08/16
WKA NO. 11071.01	

LOGS OF TEST PITS (Continued)
The Grove
Excavated on August 1st, 2016, with a Case 580 Super M excavator
Logged by: Alexander Wright
WKA No. 11071.01

TEST PIT 5

0' to 2'	Reddish brown, dry, medium dense, silty fine to coarse SAND (SM) with trace fine gravel
2' to 4'	Reddish brown, dry to moist, intensely weathered to decomposed granodioritic ROCK (RX) with fine to coarse sand
4' to 7'	Brown/grayish brown, dry to moist, dense to very dense, intensely weathered to decomposed granodioritic ROCK (RX) with fine to coarse sand

Bottom of test pit at 7 feet below existing ground surface
Groundwater was not encountered

TEST PIT 6

0' to 2'	Reddish brown, dry, medium dense, variably cemented, silty fine to coarse SAND (SM) with trace amounts of angular fine gravel
2' to 5'	Reddish brown/brown, dry to moist, dense to very dense, intensely weathered to decomposed granodioritic ROCK (RX) with fine to coarse sand

Bottom of test pit at 5½ feet below existing ground surface
Groundwater was not encountered

TEST PIT 7

0' to 2'	Reddish brown/brown, dry, medium dense, silty fine to coarse SAND
2' to 5'	Reddish brown/brown, dry to moist, dense to very dense, intensely weathered to decomposed granodioritic ROCK (RX) with fine to coarse sand
4' to 5'	Clay seam exposed along one edge of test pit

Bottom of test pit at 5 feet below existing ground surface
Groundwater was not encountered

UNIFIED SOIL CLASSIFICATION SYSTEM

MAJOR DIVISIONS		SYMBOL	CODE	TYPICAL NAMES
COARSE GRAINED SOILS (More than 50% of soil > no. 200 sieve size)	GRAVELS (More than 50% of coarse fraction > no. 4 sieve size)	GW		Well graded gravels or gravel - sand mixtures, little or no fines
		GP		Poorly graded gravels or gravel - sand mixtures, little or no fines
		GM		Silty gravels, gravel - sand - silt mixtures
		GC		Clayey gravels, gravel - sand - clay mixtures
	SANDS (50% or more of coarse fraction < no. 4 sieve size)	SW		Well graded sands or gravelly sands, little or no fines
		SP		Poorly graded sands or gravelly sands, little or no fines
		SM		Silty sands, sand - silt mixtures
		SC		Clayey sands, sand - clay mixtures
FINE GRAINED SOILS (50% or more of soil < no. 200 sieve size)	SILTS & CLAYS LL < 50	ML		Inorganic silts and very fine sands, rock flour, silty or clayey fine sands or clayey silts with slight plasticity
		CL		Inorganic clays of low to medium plasticity, gravelly clays, sandy clays, silty clays, lean clays
		OL		Organic silts and organic silty clays of low plasticity
	SILTS & CLAYS LL ≥ 50	MH		Inorganic silts, micaceous or diatomaceous fine sandy or silty soils, elastic silts
		CH		Inorganic clays of high plasticity, fat clays
		OH		Organic clays of medium to high plasticity, organic silty clays, organic silts
HIGHLY ORGANIC SOILS		Pt		Peat and other highly organic soils
ROCK		RX		Rocks, weathered to fresh
FILL		FILL		Artificially placed fill material

OTHER SYMBOLS

= Drive Sample: 2-1/2" O.D. Modified California sampler

= Drive Sampler: no recovery

= SPT Sampler

= Initial Water Level

= Final Water Level

= Estimated or gradational material change line

= Observed material change line

Laboratory Tests

PI = Plasticity Index

EI = Expansion Index

UCC = Unconfined Compression Test

TR = Triaxial Compression Test

GR = Gradational Analysis (Sieve)

K = Permeability Test

GRAIN SIZE CLASSIFICATION

CLASSIFICATION	RANGE OF GRAIN SIZES	
	U.S. Standard Sieve Size	Grain Size in Millimeters
BOULDERS	Above 12"	Above 305
COBBLES	12" to 3"	305 to 76.2
GRAVEL	3" to No. 4	76.2 to 4.76
coarse (c)	3" to 3/4"	76.2 to 19.1
fine (f)	3/4" to No. 4	19.1 to 4.76
SAND	No. 4 to No. 200	4.76 to 0.074
coarse (c)	No. 4 to No. 10	4.76 to 2.00
medium (m)	No. 10 to No. 40	2.00 to 0.420
fine (f)	No. 40 to No. 200	0.420 to 0.074
SILT & CLAY	Below No. 200	Below 0.074

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UNIFIED SOIL CLASSIFICATION SYSTEM

THE GROVE

Loomis, California

FIGURE	11
DRAWN BY	RWO
CHECKED BY	JRY
PROJECT MGR	DRG
DATE	08/16
WKA NO.	11071.01

APPENDICES

APPENDIX A
General Project Information, Laboratory Testing and Results

APPENDIX A

A. GENERAL INFORMATION

The performance of a geotechnical engineering investigation for *The Grove* residential development, located at 3342 Humphrey Road in Loomis, California, was authorized by Mr. Robert Sprague of Mandarich Developments on July 15, 2016. Authorization was for an investigation as described in our proposal letter dated July 7, 2016, sent to our client, Mandarich Developments, who's mailing address is 4740 Rocklin Road in Rocklin, California 95677; telephone (916) 825-8104.

The Civil Engineering consultant for this project is Meredith Engineering, whose mailing address is PO Box 4391, El Dorado Hills, California 95762; telephone (530) 676-7526.

B. FIELD EXPLORATION

We observed the site conditions and performed subsurface exploration at the site on August 2, 2016. Field explorations consisted of six soil borings and seven test pits, to explore the subsurface soil and rock conditions to depths of approximately 5 to 12.5 feet below the existing ground surface. Soils exposed in the trenches and recovered from the borings were classified as required by the 2013 CBC in accordance with ASTM D2487. Soil descriptions are reported using a modified naming system.

Soil borings were completed by V&W Drilling using a CME 75 truck-mounted drilling rig with a 6-inch diameter solid flight auger drill. Relatively undisturbed soil samples were obtained at various intervals in each boring using a Modified California Sampler with liners (1.875 inch inside diameter, 2.5 inch outside diameter).

Test pits were completed by Ron Tilford Backhoe using a Case 580 Super M excavator and an 18-inch bucket. Bulk samples were obtained from disturbed excavated soil cuttings at the time of excavation. Caving and sloughing of soil in the test pit sidewalls was not observed during excavation, with test pits generally stable during the excavation process. We note that excavation occurred during the summer months, and that excavations were not open long term or subject to wetting. At the completion of exploration activities, the borings were backfilled with soil cuttings, and compacted using a sheepsfoot compaction wheel. We note that only moderate compaction was achieved due to lack of moisture, and that test pits should be located and re-compacted as needed during construction.

The approximate locations of the soil borings and test pits completed as part of the soil exploration program are provided on Figure 2. Descriptions of the soils encountered in the test borings are presented on Figures 3 to 8, and logs of test pits are presented on Figure 9 and 10. An explanation of the Unified Soil Classification System symbols used in the soil descriptions is presented on Figure 11.

C. LABORATORY TESTING

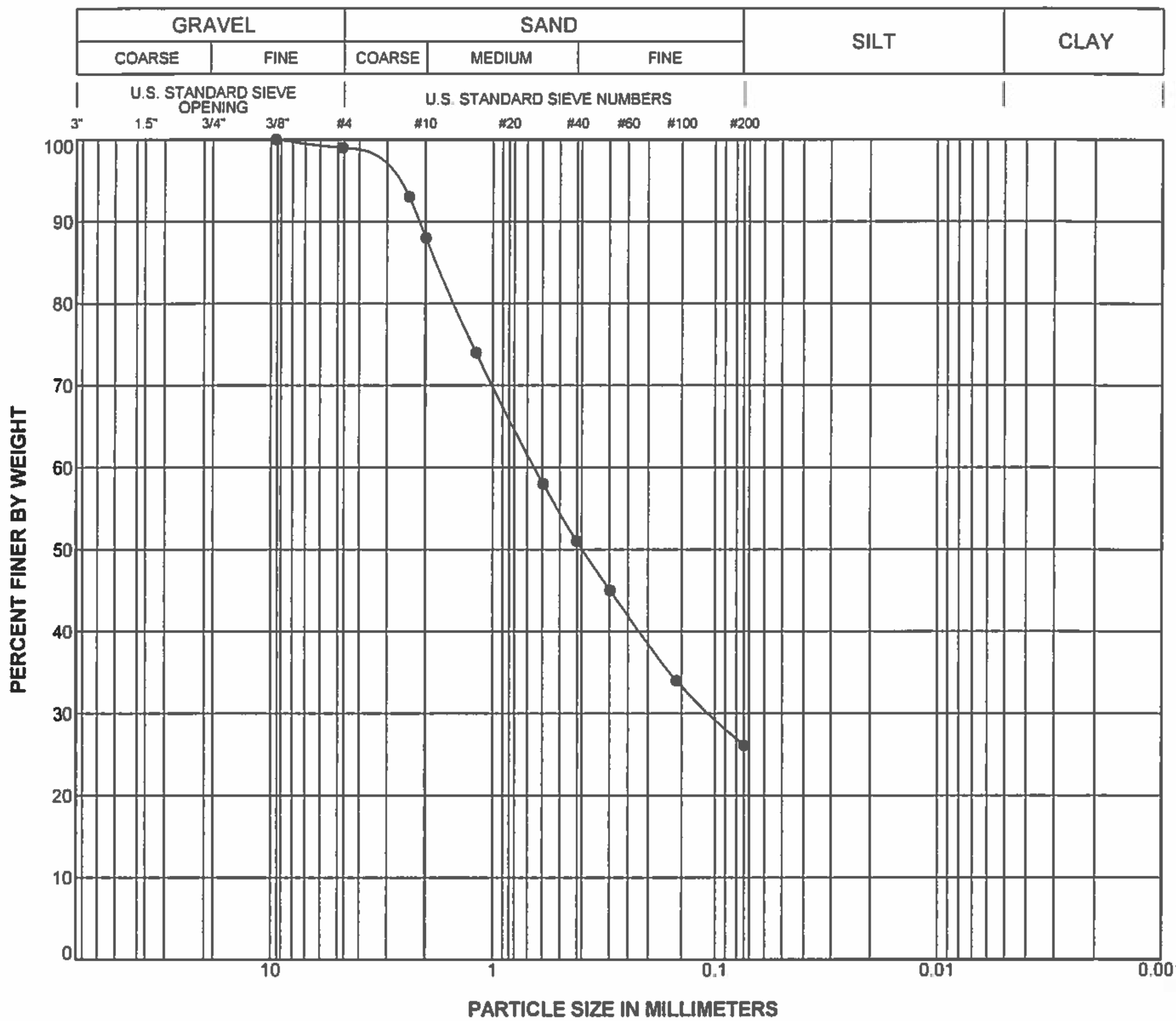
Selected soil samples were tested to determine dry unit weight (ASTM D 2937) and natural moisture content (ASTM D 4643). The results of these tests are included on the boring logs at the depth each sample was obtained.

A representative sample of near-surface soils was tested for grain-size distribution (ASTM C136). The results of the gradation tests are contained in Figure A1.

A sample of near-surface soil considered to be representative of the on-site soils was subjected to Resistance "R" value testing (CT 301). The test results are presented in Figure A2.

A sample of the near-surface soil was tested to evaluate shear strengths using a consolidated undrained with pore pressures triaxial test, commonly known as a CUPP triaxial test (ASTM D4767) with results presented in Figure A3.

A sample of the near-surface soil was submitted to Sunland Analytical to determine the soil pH and minimum resistivity (California Test 643), Sulfate concentration (California Test 417, ASTM D516) and Chloride concentration (California Test 422). The results of these tests are presented in Figure A4.



Boring Number	Sample Number	USCS	Depth (feet)	Symbol	LL	PI	Classification
TP3	B1	SM	0' to 2'	●			Dark reddish brown silty fine to coarse SAND (SM)

GRAIN SIZE 11071.01 - THE GROVE.GPJ WKA.GDT 8/26/16 2:59 PM

PARTICLE SIZE DISTRIBUTION

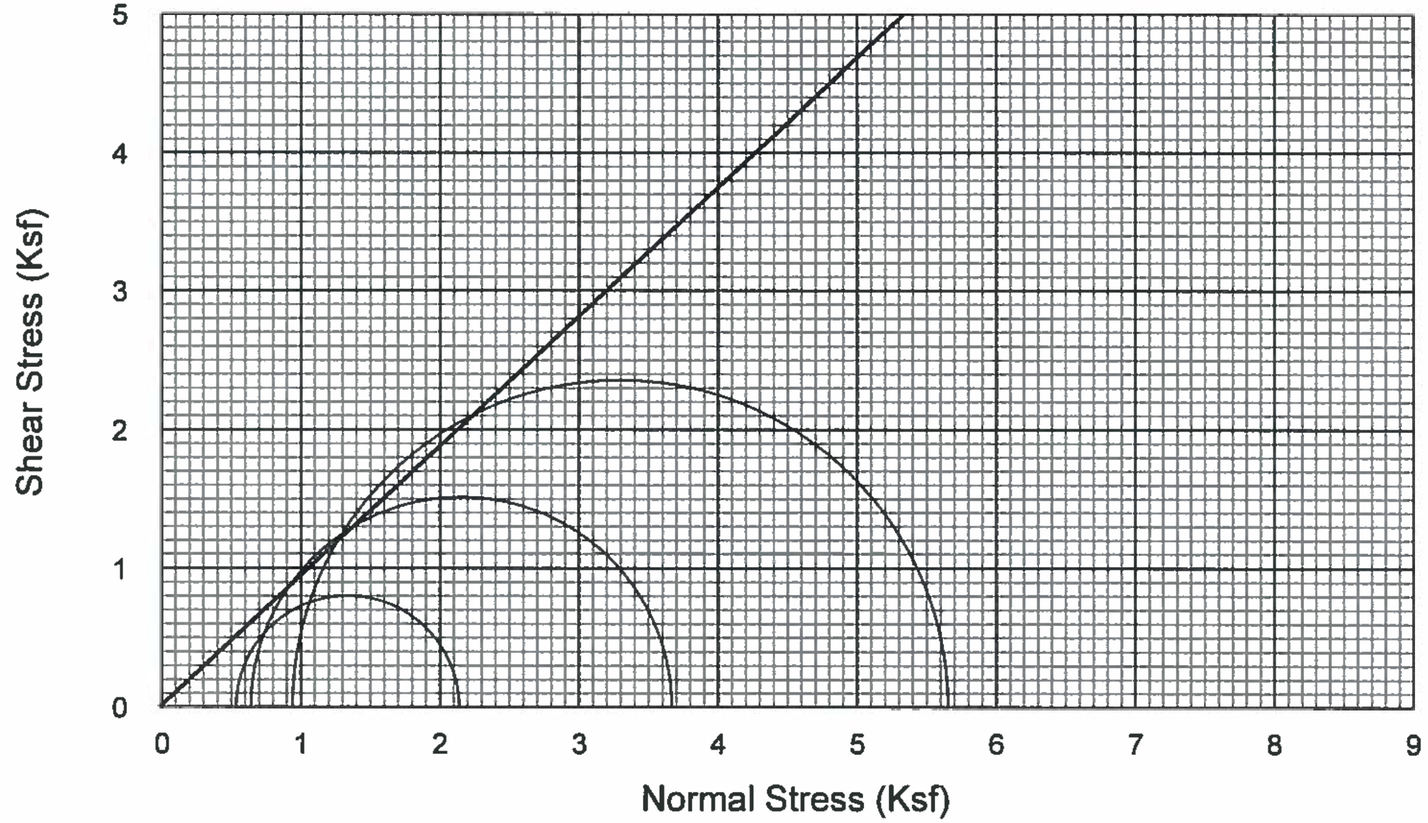
Project: The Grove
WKA No. 11071.01

FIGURE A1

Wallace Kuhl & ASSOCIATES

TRIAXIAL COMPRESSION TEST

ASTM D4767



SAMPLE NO. : D3-1I

SAMPLE CONDITION : Undisturbed

SAMPLE DESCRIPTION : Light brown silty fine to coarse SAND (SM)

DRY DENSITY (PCF) : 115
INITIAL MOISTURE (%) : 5.81
FINAL MOISTURE (%) : 15.6

ANGLE OF INTERNAL FRICTION (ø) : 43°
COHESION (PSF) : 00

WallaceKuhl & ASSOCIATES

TRIAXIAL COMPRESSION TEST RESULTS

THE GROVE

Plumas Lake, California

FIGURE	A2
DRAWN BY	RWO
CHECKED BY	ADW
PROJECT MGR	DRG
DATE	08/16
WKA NO. 11071.01	

RESISTANCE VALUE TEST RESULTS

(California Test 301)

MATERIAL DESCRIPTION: Dark reddish brown silty fine to coarse SAND (SM)

LOCATION: TP3 B1 (0' - 2')

Specimen No.	Dry Unit Weight (pcf)	Moisture @ Compaction (%)	Exudation Pressure (psi)	Expansion (dial, inches x 1000)	Expansion (psf)	R Value
4	125	10.4	152	1	4	28
5	128	9.2	385	3	13	76
6	126	9.7	222	2	9	60

R-Value at 300 psi exudation pressure = **69**

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RESISTANCE VALUE TEST RESULTS

The Grove

Plumas Lake, California

FIGURE	A3
DRAWN BY	RWO
CHECKED BY	ADW
PROJECT MGR	DRG
DATE	08/16
WKA NO.	11071.01

Sunland Analytical

11419 Sunrise Gold Circle, #10
Rancho Cordova, CA 95742
(916) 852-8557

Date Reported 08/05/2016
Date Submitted 08/02/2016

To: Joey Ybarra
Wallace-Kuhl & Assoc.
3050 Industrial Blvd
West Sacramento, CA 95691

From: Gene Oliphant, Ph.D. \ Randy Horney RH
General Manager \ Lab Manager

The reported analysis was requested for the following:
Location : 11071.01 Site ID : P3 51 0-1FT.
Thank you for your business.

* For future reference to this analysis please use SUN # 72492-151403.

Extractable Sulfate in Water

TYPE OF TEST	RESULTS	UNITS
Sulfate-SO4	5.50	mg/kg

ASTM D-516 from sat.paste extract-reported based on dry wt.

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CORROSION TEST RESULTS

THE GROVE

Plumas Lake, California

FIGURE	A4
DRAWN BY	RWO
CHECKED BY	ADW
PROJECT MGR	DRG
DATE	08/16
WKA NO.	11071.01

Sunland Analytical

11419 Sunrise Gold Circle, #10
Rancho Cordova, CA 95742
(916) 852-8557

Date Reported 08/05/2016
Date Submitted 08/02/2016

To: Joey Ybarra
Wallace-Kuhl & Assoc.
3050 Industrial Blvd
West Sacramento, CA 95691

From: Gene Oliphant, Ph.D. \ Randy Horney
General Manager \ Lab Manager

The reported analysis was requested for the following location:
Location : 11071.01 Site ID : P3 51 0-1FT.
Thank you for your business.

* For future reference to this analysis please use SUN # 72492-151402.

EVALUATION FOR SOIL CORROSION

Soil pH	5.44	
Minimum Resistivity	7.50 ohm-cm (x1000)	
Chloride	5.1 ppm	00.00051 %
Sulfate	5.1 ppm	00.00051 %

METHODS
pH and Min.Resistivity CA DOT Test #643
Sulfate CA DOT Test #417, Chloride CA DOT Test #422

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CORROSION TEST RESULTS

THE GROVE

Plumas Lake, California

FIGURE	A5
DRAWN BY	RWO
CHECKED BY	ADW
PROJECT MGR	DRG
DATE	08/16
WKA NO.	11071.01

APPENDIX B
Earthwork Specifications

EARTHWORK SPECIFICATIONS
THE GROVE RESIDENTIAL SUBDIVISION
Humphries Road, Loomis, California
WKA No. 11071.01

GEOTECHNICAL ENGINEERING REPORT

A Geotechnical Engineering Report (WKA No. 11071.01, dated September 9, 2016) has been prepared under the supervision of a California Licensed Geotechnical Engineer for this project by Wallace - Kuhl & Associates of West Sacramento, California; telephone (916) 372-1434. Where specific reference is made to "Geotechnical Engineer," this designation shall be understood to include the engineer of record retained by the Owner to provide observation and testing services during construction, or his or her designated representative.

SEASONAL LIMITS

Fill material shall not be placed, spread or rolled during unfavorable weather conditions. When heavy rains interrupt the work, fill operations shall not be resumed until field tests indicate that the moisture contents of the subgrade and fill materials are satisfactory.

FILL MATERIALS

a) All engineered fill shall be of approved local materials from required excavations, supplemented by imported fill, if necessary. On site fill materials shall be derived from processed approved local materials defined as local coarse grained soils as defined by ASTM D2487, clean from rubble, rubbish, organics, and vegetation, and shall be approved by the Geotechnical Engineer prior to use. Clods or rocks exceeding three inches (3") in final size shall not be allowed.

b) Imported fill materials and shall be compactable, coarse grained granular soils as defined by ASTM D2487 with a Plasticity Index as defined by ASTM D4318 of fifteen (15) or less; an Expansion Index as defined by ASTM D4829 of twenty (20) or less; and, and free of significant quantities of particles greater than three inch (3") maximum particle size.

c) Class 2 aggregate base shall conform to the Caltrans Standard Specifications, latest edition, and must be approved by the geotechnical engineer prior to use on site.

CLEARING, GRUBBING AND PREPARING BUILDING PADS

a) All vegetation, brush, debris, and other items encountered during site work and deemed unacceptable by the Geotechnical Engineer, shall be removed and disposed of so as to leave the disturbed areas with a neat and finished appearance, clean from unsightly debris. Excavations and depressions resulting from the removal of such items, as determined by the Geotechnical Engineer, shall be cleaned out to firm, undisturbed soils and backfilled with suitable materials in accordance with these specifications.

b) All areas to receive engineered fill shall be prepared by scarification to a depth of at least twelve inches (12"), moisture conditioned to at least the optimum moisture content, and uniformly compacted to at least ninety percent (90%) of the maximum dry density as determined by ASTM D1557. Where weathered or hard rock is exposed, as identified by the geotechnical engineer, scarification and compaction may be waived on approval by the geotechnical engineer.

c) The building pads and all areas to receive concrete slabs and driveways, including the area contained within five feet (5') horizontally of the building foundations and slabs shall be ripped and cross-ripped to a depth of at least twelve inches (12"), moisture conditioned to at least the optimum moisture content, and uniformly compacted to at least ninety percent (90%) of the maximum dry density as determined by ASTM D1557, regardless of whether the grade is made by excavation, filling, or remains near existing grade.

d) Where weathered or hard rock is exposed, as identified by the geotechnical engineer, scarification and compaction may be waived on approval by the geotechnical engineer.

e) Compaction operations shall be performed in the presence of the Geotechnical Engineer who will evaluate the performance of the materials under compactive load. Unstable soil deposits, as determined by the Geotechnical Engineer, shall be excavated to expose a firm base and grades restored with select or imported fill.

PLACING, SPREADING AND COMPACTING FILL MATERIAL

a) Engineered fill material shall be placed in layers, which when compacted shall not exceed six inches (6") in thickness, unless otherwise approved by the Geotechnical Engineer. Each layer shall be spread evenly, thoroughly mixed and compacted to not less than ninety percent (90%) of the maximum dry density as determined by ASTM D1557.

b) Engineered fills greater than five feet (5') in vertical thickness shall be placed in lifts, moisture conditioned to at least the optimum moisture content and compacted to not less than ninety-five percent (95%) of the maximum dry density as determined by ASTM D1557.

c) Compaction shall be undertaken using appropriate compaction equipment capable of achieving the specified density, and shall be accomplished while the fill material is at a moisture content of at least the optimum moisture content. Appropriate compaction equipment is defined as equipment consistent with the recommendations of the United State Army Corp of Engineers Naval Facilities Engineering Command Design Manual 7.02 Table 5, or as otherwise approved by the Geotechnical Engineer.

FINAL SUBGRADE PREPARATION

a) The upper twelve inches (12") of final building pad and slab-on-grade subgrades shall be brought to a uniform moisture content of at least the optimum moisture content and shall be uniformly compacted to not less than ninety percent (90%) as determined by the ASTM D1557 Test Method, regardless of whether final subgrade elevations are attained by filling, excavation, or are left near existing grades

b) The upper six inches (6") of final street subgrades shall be brought to a uniform moisture content of at least the optimum moisture content and shall be uniformly compacted to not less than ninety percent (90%) as determined by the ASTM D1557 Test Method, regardless of whether final subgrade elevations are attained by filling, excavation, or are left at existing grades.

c) Where weathered or hard rock is exposed at the finished soil subgrade elevation, as identified by the geotechnical engineer, scarification and compaction may be waived on approval by the geotechnical engineer.

d) Aggregate base material placed beneath streets shall conform to the project plans and shall be composed of Class 2 Aggregate Base and shall be compacted to not less than ninety-five percent (95%) of the maximum dry unit weight, at not less than the optimum moisture content, as determined by ASTM D1557.

UTILITY TRENCH BACKFILL

a) Pipe zone backfill, defined as the zone extending from six (6) inches below the pipe to twelve (12) inches above the pipe, shall conform to the pipe manufactures recommendations and the applicable governing agencies standards. Trench zone backfill consisting of imported or engineered fill should extend from the top of the pipe zone to a point twelve (12) inches below finished grade.
b) Final zone backfill in the upper twelve (12) inches shall conform to the compaction standards for final subgrade preparation.
c) Trench zone and final zone backfill shall be compacted to not less than ninety percent (90%) of the maximum dry density as determined by ASTM D1557, except within street areas where it shall be in conformance with the applicable city and/or county requirements, and final zone backfill shall be of imported fill compacted to not less than ninety-five percent (95%) of the maximum dry density as determined by ASTM D1557.

TESTING AND OBSERVATION

a. All grading operations shall be tested and observed by the Geotechnical Engineer, who is serving as the representative of the Owner.

b. Earthwork shall not be performed without prior notification and approval of the Geotechnical Engineer. The Contractor shall notify the Geotechnical Engineer at least two (2) working days prior to commencement of any aspect of the site earthwork.

c. If the Contractor should fail to meet the technical or design requirements embodied in this document and on the applicable plans, he shall make the necessary readjustments until all work is deemed satisfactory, as determined by the Geotechnical Engineer and the Project Engineer.

d. No deviation from these specifications shall be made except upon written approval of the Geotechnical Engineer.

/